Geotechnical Exploration and Evaluation
Bay Pines Research Building
Bay Pines, Florida

September 29, 2011
Project No. H4105204

NODARSE
A Terracon COMPANY

Ms. Pamela Ringling Caffery
VA Contracting Representative
RDC/John Poe Architects
524 Fernwood Drive
Altamonte Springs, Florida 32701

Report of Geotechnical Exploration and Evaluation
Proposed Bay Pines VA Research Building
Bay Pines, Florida

Dear Ms. Caffery:

Nodarse & Associates, a Terracon Company is pleased to present this report of geotechnical engineering exploration and evaluation for the referenced site. The purpose of our exploration was to explore soil and groundwater conditions at the site in order to provide recommendations for a foundation system. This report describes the field exploration performed, the soil and groundwater conditions encountered, and our evaluation different foundation systems regarding the proposed construction.

PROJECT DESCRIPTION

The proposed research building is to be constructed as a stand-alone addition to the west of Building 23 at the Bay Pines VA Medical Center. The Bay Pines Medical Center is located in southwest Pinellas County, west of the City of St. Petersburg. More specifically the address of the Medical Center is 1000 Bay Pines Boulevard, Bay Pines, Florida. **Figure 1** in the **Appendix** shows the location of the building utilizing the "Seminole, Florida" USGS quadrangle map.

The proposed building will be a two story structure with a footprint of about 4,200 square feet. Structural support is to be provided by a cast-in-place concrete pan-joist system utilizing load bearing walls and columns. We have been provided column loads ranging in magnitude from 150 kips to approaching 700 kips. We understand that the loads for the proposed structure are significantly higher than what would be expected in a typical two story building because this structure is to be designed to resist a progressive collapse. The finished grade of the 1st floor will be several feet above the existing grade and there will be a crawl space under the 1st floor. The building will be constructed in a flood zone.

PURPOSE AND SCOPE

The purpose of our exploration was to evaluate soil and groundwater conditions at the site and to provide a basis for geotechnical engineering recommendations regarding the proposed construction. For the stated purpose, the following scope of services was performed:

- Notified Sunshine of Florida for utility locates and coordinated the boring locations with VA Bay Pines personnel to avoid striking underground utilities.

- Performed three standard penetration test (SPT) borings to a depth of 50 feet each. Each boring was grouted upon completion.

- Geotechnical engineering and support services included the continual involvement by geotechnical engineers to direct the field work, data reduction analysis, and evaluation and preparation of this geotechnical report with design level recommendations.

SITE CONDITIONS

Based on review of aerial photography, USGS Quadrangle Map containing the site provided topographic information and observations made during the field exploration, the following site conditions are offered:

- According to the USGS topographic map, the natural ground surface elevation of the site is approximately +10 feet NGVD.
- The Medical Center is bordered by Boca Ciega Bay. The area in the vicinity of Building 23 includes paved parking and landscaped areas and is relatively flat.

SOIL CONDITIONS

Figure 2 in the **Appendix** presents the NRCS Soil Survey for Pinellas County, Florida. The following natural surficial soil type is shown to be present at the building site:

- **Immokalee Soils and Urban Land (13).** This soil is nearly level and poorly drained. In its natural state, the seasonal high groundwater table is typically 6 to 18 inches below the existing ground surface.

Field Exploration

Borings were marked in the field by N&A personel by measuring from existing land marks. The approximate locations of the soil borings are presented on **Figure 3** in the **Appendix**.

Standard Penetration Tests (SPT) were performed continuously in the SPT borings from the existing ground surface to a depth of 10 feet and at 5 foot depth intervals thereafter. Each sample was removed from the sampler in the field and was examined and visually classified by an engineering technician. Representative portions of each sample were packaged and sealed for transportation to our laboratory for further examination and visual classification. Water levels were measured in the boreholes at the time of our field exploration to evaluate the depth to groundwater. The boreholes were then grouted after completion.

Adjacent to the SPT boring profiles are the "N" values. These "N" values are the number of hammer blows required to advance the split spoon sampler a distance of 12 inches. The "N" values have been empirically correlated with various soil properties and are considered to be indicative of the relative density of cohesionless soils and consistency of cohesive soils.

Subsurface Conditions

In general the borings encountered a thin surficial layer of top soil (Strata 1) underlain by loose to dense slightly silty fine sand (Strata 2) to a depth of 12 feet. Then, an approximately 10 foot layer of very loose to very dense clayey sand and calcareous clayey sand (Stratum 3) was encountered to approximately 22 feet underlain by an approximately 5 foot thick layer of stiff to very stiff desiccated silty clay (Stratum 4). From 27 feet until the termination of the borings layers of clayey sand (Stratum 5), silty sand (Stratum 6), and silty clay (Stratum 4), slightly silty fine sand (Stratum 8), weathered limestone (Stratum 7) and limestone (Stratum 7) were encountered. The thickness and depth of lower layers were highly variable from location to location.

The Standard Penetration Test resistances (N-values) recorded in the sand layers (Strata 1, 2, 3, 5, 6 and 8) ranged from 5 blows per foot (BPF) to 50 blows for 3 inches of penetration. The blow counts in the clay layers (Stratum 4) ranged from 10 BPF to 50 blows per 2 inches of penetration. The N-values in the limestone formation ranged from 32 BPF to 50 blows with ½ inch of penetration.

For details at individual boring locations and soil boring profiles, refer to boring profiles on **Figures 3 and 4** in the **Appendix**.

Groundwater Table

The groundwater table (shallow non-artesian) was measured in borings at a depth ranging from 2 to 2.8 feet below the existing ground surface.

Groundwater levels regularly fluctuate throughout the year, and therefore, may be different at other times. Groundwater levels at the site will also vary due to fluctuations in the amount of local rainfall, irrigation or site development. Based on review of the Pinellas County Soil Survey, topographic information, the apparent presence of fill soil and our experience with similar sites, the typical seasonal high groundwater level at the boring locations is expected to be about 1 to 1.5 feet below existing grade.

It should be understood that these estimated seasonal high groundwater levels are based on the prevailing groundwater level at the time of this study and other published historical data which may be available. It does not imply or guarantee that under certain circumstances of high rainfall conditions or alterations to this or adjoining sites or significant changes in the operating characteristics of adjoining drainage features, that groundwater levels can not be higher than the estimate given above.

Observed groundwater levels are shown adjacent to the soil boring profiles, on **Figure 4** in the **Appendix**.

RECOMMENDATIONS AND EVALUATIONS

The following conclusions and recommendations are based on the project characteristics previously described, the data obtained in our field exploration and our experience with similar subsurface conditions and construction types.

We have evaluated several foundation systems to support the proposed structures. The following foundation systems were evaluated:

- Shallow Foundations
- Auger Cast Piles
- Precast Concrete Piles
- Drilled Shafts
- Ductile Iron Piles

Shallow Foundations Recommendations

Based on the magnitude of the loads induced on the soils from the research building (Column loads near 700 kips), conventional shallow foundations would not be an appropriate foundation system for this structure. Bearing these buildings on shallow foundations would lead to excess settlement due to the large loads. In addition, it is also our understanding that the VA Bay Pines Facility could be subject damage during a significant storm event where soil erosion could occur and therefore a deep foundation system would be required.

Augercast Piles

Augercast piles should be considered as a viable foundation alternative for this project. Augercast pile installation involves drilling a hollow stem flighted auger into the soils. Once the auger reaches its termination depth, grout is pumped through the hollow stem and the auger is removed and a small diameter reinforcing cage or bar is inserted. The piles would be embedded in the hard clays and limestone rock formation encountered in the borings. Augercast pile installation is typically much quieter and produces fewer vibrations than some other foundation installation methods.

We have analyzed 14, 16 and 18 inch diameter augercast piles by modeling them as drilled shafts in the FDOT Software FBDeep. Our analysis is based on the borings performed during this exploration. We have estimated the allowable skin friction of the piles based on a factor of safety of 2. End bearing resistance was neglected in our analysis. The results of the augercast pile analysis are presented in the **Augercast Pile Capacity Tables** located in the **Appendix**. We have outlined the allowable resistance with depth obtained by taking the worst case scenario of the three borings. This information is presented below:

Table 1: Allowable Skin Friction vs Depth

Allowable Side Friction (kips)	Pile Tip Depth Below Existing Ground Surface (ft)		
	Augercast Pile Diameter (inches)		
	14	16	18
50	32	31	30
75	37	36	34
100	45	40	38
125	*	*	44
150	*	*	*

*Capacity not obtained within depth of borings.

Precast Concrete Piles

We have considered precast driven piles as a possible foundation alternative. The installation of driven piles creates as significant amount of noise that can be a nuisance to the existing hospital building. Additionally, pile driving operations typically create a significant amount of soil vibrations that may cause damage to surrounding structures.

The computer model **FBDeep** was used to evaluate estimated ultimate capacities for the 14 and 18-inch square prestressed concrete pile sections. This analysis is known to be conservative when driving into hard layers. The input soil parameters were obtained from the SPT borings performed. A factor of safety of 2 was used in our analysis. As with the augercast analysis, the worst case scenario has been used and the piles may achieve the indicated capacity at a shallower depth. The allowable capacities versus pile tip elevations for the 14 and 18-inch square concrete piles are shown in Table 2 below:

Allowable Capacity (kips)	**Pile Tip Depth from Existing Ground Surface (ft)**	
	Driven Pile Width (inches)	
	14	**18**
100	27	23
150	*	27
200	*	*

*Capacity not obtained within depth of borings.

Based on the FDOT Structures Design Guidelines, maximum pile driving resistance for a 14-inch prestressed concrete pile should not exceed 400 kips and a maximum pile driving resistance for a 18-inch prestressed concrete pile should not exceed 600 kips. Based on our subsoil exploration, dense soils encountered, and capacity curves, it is recommended that for a 14-inch pile, the resistance be limited to 300 kips; and for an 18-inch pile, the maximum resistance be limited to 500 kips. We have conducted our analysis based on the upper 45 feet of the soil profiles because our borings were terminated at 50 feet. If driven to this depth, the maximum Davisson resistance allowed for each pile may not be realized. If the driven piles are selected as the foundation alternative, deeper borings may be necessary if the maximum allowable structural capacity of the pile is to be achieved.

Drilled Shafts (Caissons)

Drilled shafts (Caissons) were also considered as a deep foundation system for the project. Drilled shafts are typically embedded in hard clays or the limestone rock formation to form a "rock socket" which will carry the load. Drilled shafts have an advantage of being able to resist high loads with a single shaft. Column loads provided to us for this building can typically be handled by a single drilled shaft while other foundations systems may require several piles per column. Like augercast piles, drilled shafts installation produces a relatively low amount of noise and vibrations.

While borings B-2 and B-3 indicated that a rock socket could be achieved, the results of boring B-1 indicated that soil conditions for a rock socket are not favorable until the limestone layer at about 37 feet is reached. Given that the borings were terminated at 50 feet, there was insufficient information to allow for a drilled shaft design to be performed. Should the borings be deepened, then parameters for a drilled shaft foundation system could be developed.

Note that this variable soil condition is not unusual for this area of Florida. As a result, when drilled shafts are utilized for foundation support, pilot holes are drilled at each shaft location and the tip elevation of each shaft is determined at that time. Thus, the required founding elevation for each shaft is based not only on the imposed loads, but also the conditions at each shaft location.

Ductile Iron Piles

We have considered ductile iron piles (DIP) as a foundation alternative for this project. These piles consist of ductile iron pipe piles that are driven utilizing a high frequency pile hammer. The pile length is easily adjustable because additional pipe is added as necessary. The pipes can be backfilled with concrete to provide more structural capacity. Ductile iron piles have the advantage of being relatively quick to install and do not create a significant amount of vibrations. DIP's are typically driven to refusal. We expect and allowable capacity on the order of 75 to 100 kips per pile.

Load Testing

Based on the foundation alternative selected, we recommend performing at least one load test on a completed pile. This load test may be either dynamic or static and should be selected based on the foundation alternative selected.

GENERAL SITE PREPARATION

Initial Site Preparation

Prior to the initiation of the installation of a deep foundation system, we recommend that the site be brought to rough final grade. This is because, after the foundations are installed, it will likely be difficult to maneuver earthmoving equipment within the building area.

Areas that will support footings, pile caps, floors, pavements or new engineered fill must be properly prepared. All topsoil and unsuitable materials should be removed to a distance of 5 feet beyond the perimeter of construction. Unsuitable materials include topsoil, asphaltic concrete, concrete slabs and pavement, any soft unstable material and miscellaneous (non-soil) fill. Removal of trees and shrubs should include removal of the root system down to finger-sized roots.

Prior to construction of slabs on grade, the geotechnical engineer should evaluate the exposed subgrade. The evaluation should include proofrolling of the exposed subgrade to aid in detecting buried utilities or debris not otherwise found. The proofrolling operation should be conducted under the direct supervision of the geotechnical engineer utilizing either a senior engineering technician or engineer. If unsuitable materials are disclosed, the geotechnical engineer would recommend appropriate remedial measures at that time. The proofrolling can consist of rolling all areas with ten passes of a vibratory roller with a minimum static weight of 20,000 pounds. The latter five passes should be at right angles to previous passes. Any areas that yield excessively under the proofrolling operations should be removed and replaced by a suitable fill material as noted later in this report.
Proofrolling should be continued until the soils in the building area have achieved a minimum density of 95 percent of the maximum dry density as determined by ASTM D 1557 (Modified Proctor) to a minimum depth of 24 inches below the ground surface. In-place density tests should be conducted by a qualified geotechnical engineering technician working under the direction of a licensed geotechnical engineer.

Care should be exercised during grading and fill placement operations. The combination of heavy construction equipment traffic and excess surface moisture can cause pumping and deterioration of the near surface soils. The severity of this potential problem depends to a great extent on the weather conditions prevailing during construction. The contractor should exercise discretion when selecting equipment sizes and also make a concerted effort to control surface water while the subgrade soils are exposed. If such problems do arise, the operations in the affected area should be halted and the geotechnical engineer contacted to evaluate the condition.

Fill Placement

After the site has been prepared as described above and accepted by the geotechnical engineer, fill required to bring the site to final grade may be placed and properly compacted as follows:

- Fill should be inorganic, non-plastic, granular soil (clean sands). Preferably it should have less than 12 percent passing a No. 200 sieve. The suitability of specific soils as fill material would be based on the results from classification and compaction tests and subject to approval of the geotechnical engineer. Soils found in the initial 12 feet of each boring (excluding the surficial topsoil) are suitable for use as fill.

- The fill should be placed in level lifts not to exceed 12 inches loose thickness if a large roller or heavy equipment is used to compact the fill.

- The fill should be compacted to a minimum of 95 percent of the soil's modified Proctor maximum dry density as determined by ASTM Specification D-1557.

- In-place density tests should be performed on each lift by an experienced engineering technician working under the direction of a licensed geotechnical engineer to verify that the recommended degree of compaction has been achieved. We suggest a minimum testing frequency of one test per lift per 2,500 square feet of area within the building areas.

- Fill should extend a minimum of 5 feet beyond building lines to prevent possible erosion or undermining of footing bearing soils. Further, fill slopes should not be steeper than 2 horizontal to 1 vertical (2H:1V).

- Fill placed in utility trenches and adjacent to footings beneath slabs on grade should also be properly placed and compacted to the specifications stated above. However, in these restricted working areas, compaction should be accomplished with lightweight, hand-guided compaction equipment and lift thicknesses should be limited to a maximum of 6 inches loose thickness.

Temporary Dewatering

Dewatering may be needed to facilitate excavations and compaction operations deeper than 1 foot. The necessity for dewatering will be dependent on the depth of excavation below existing grade and the groundwater levels at the time of construction. Actual dewatering means and methods should be left up to a contractor experienced in installation and operation of dewatering systems. However, dewatering for general earthwork operations can probably be accomplished at this site by a system of temporary drainage ditches graded to drain to sumps which can be pumped sufficiently to maintain water levels at the ditch bottoms. Of particular concern would be where dewatering operations could lower the groundwater under the existing structures, leading to some settlement of that structure. The contractor should provide a dewatering plan for review and approval by the engineer prior to the installation of the dewatering systems.

CLOSURE

Our professional services have been performed, our findings obtained, and our recommendations prepared in accordance with generally accepted geotechnical engineering principles and practices. This warranty is in lieu of all other warranties, either expressed or implied. **N&A** is not responsible for the independent conclusions, opinions or recommendations made by others based on the field exploration and laboratory test data presented in this report.

N&A appreciates the opportunity to be of service to you on this project and trusts this report meets your immediate needs. If you should have any questions regarding this report or if we may be of further assistance, please do not hesitate to contact us.

Sincerely,

NODARSE, a Terracon Company

Brad M Johnson, E.I.
Project Engineer

Stephen C. Knauss, P.E.
Senior Project Engineer
FL Registration No. 28202

STEPHEN CHARLES KNAUSS
LICENSE
No. 28202
STATE OF
FLORIDA
PROFESSIONAL ENGINEER

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1 - File

Appendix: Figure 1 – USGS Quadrangle Map
Figure 2 – USDA Soils Map
Figure 3 – Boring Location Plan
Figure 4 – Soil Boring Profiles

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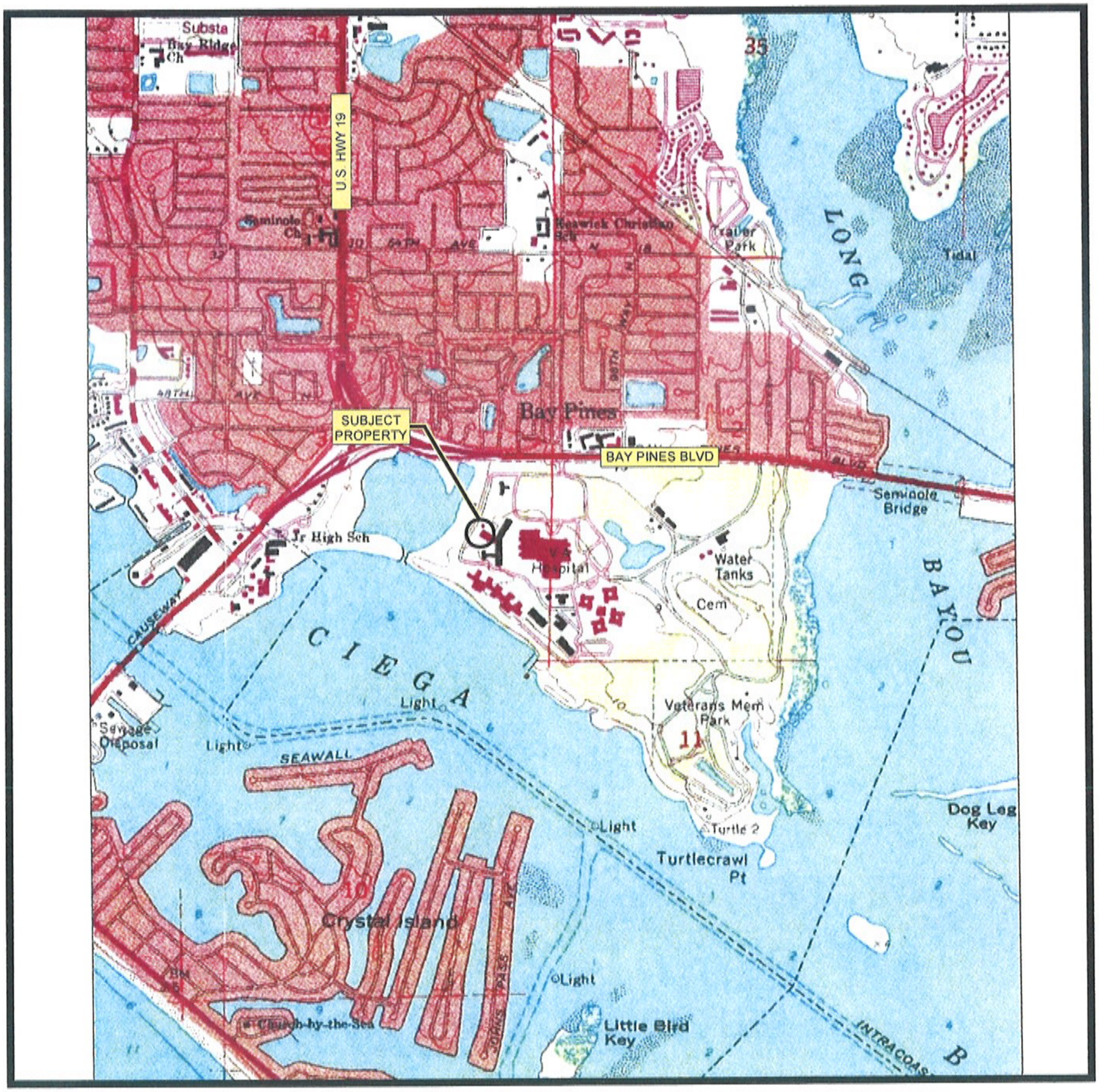
APPENDIX



REFERENCE: U.S.G.S. "SEMINOLE, FLORIDA" QUADRANGLE MAP
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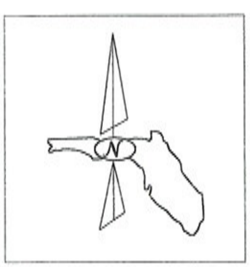
TOWNSHIP: 31 SOUTH
RANGE: 15 EAST

ISSUED: 1988
REVISED: 1987



U.S.G.S. QUADRANGLE MAP
BAY PINES VA
RESEARCH BUILDING
PINELLAS COUNTY, FLORIDA

DRAWN: TMB	NODARSE & ASSOCIATES, INC.
CHKD: BMJ	
SCALE: 1"=2000'	
DATE: 09-14-11	PROJ. NO: H4105204 — FIGURE: 1

Sep14, 2011-1:33pm N:\Projects\2010\H4105204\Working Files\Diagrams-Drawings-Figures\H4105204-Figure-1.dwg

REFERENCE: U.S.D.A. - SOIL SURVEY FOR PINELLAS COUNTY, FLORIDA

ISSUED: JANUARY 2010

SECTION: 2
TOWNSHIP: 31 SOUTH
RANGE: 15 EAST

SOILS MAP INDEX

13 IMMOKALEE SOILS AND URBAN LAND

U.S.D.A. - SOILS MAP
BAY PINES VA
RESEARCH BUILDING
PINELLAS COUNTY, FLORIDA

DRAWN: TMB	NODARSE & ASSOCIATES, INC.	
CHKD: BMJ		
SCALE: 1"=2000'		
DATE: 09-14-11	PROJ. NO: H4105204	FIGURE: 2

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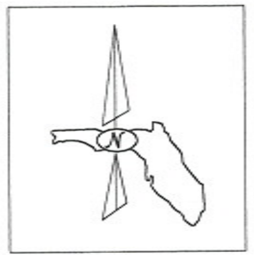

LEGEND

APPROXIMATE LOCATION OF STANDARD PENETRATION TEST BORING

BORING LOCATION PLAN
BAY PINES VA
RESEARCH BUILDING
PINELLAS COUNTY, FLORIDA

DRAWN: TMB	NODARSE & ASSOCIATES, INC.
CHKD: BMJ	
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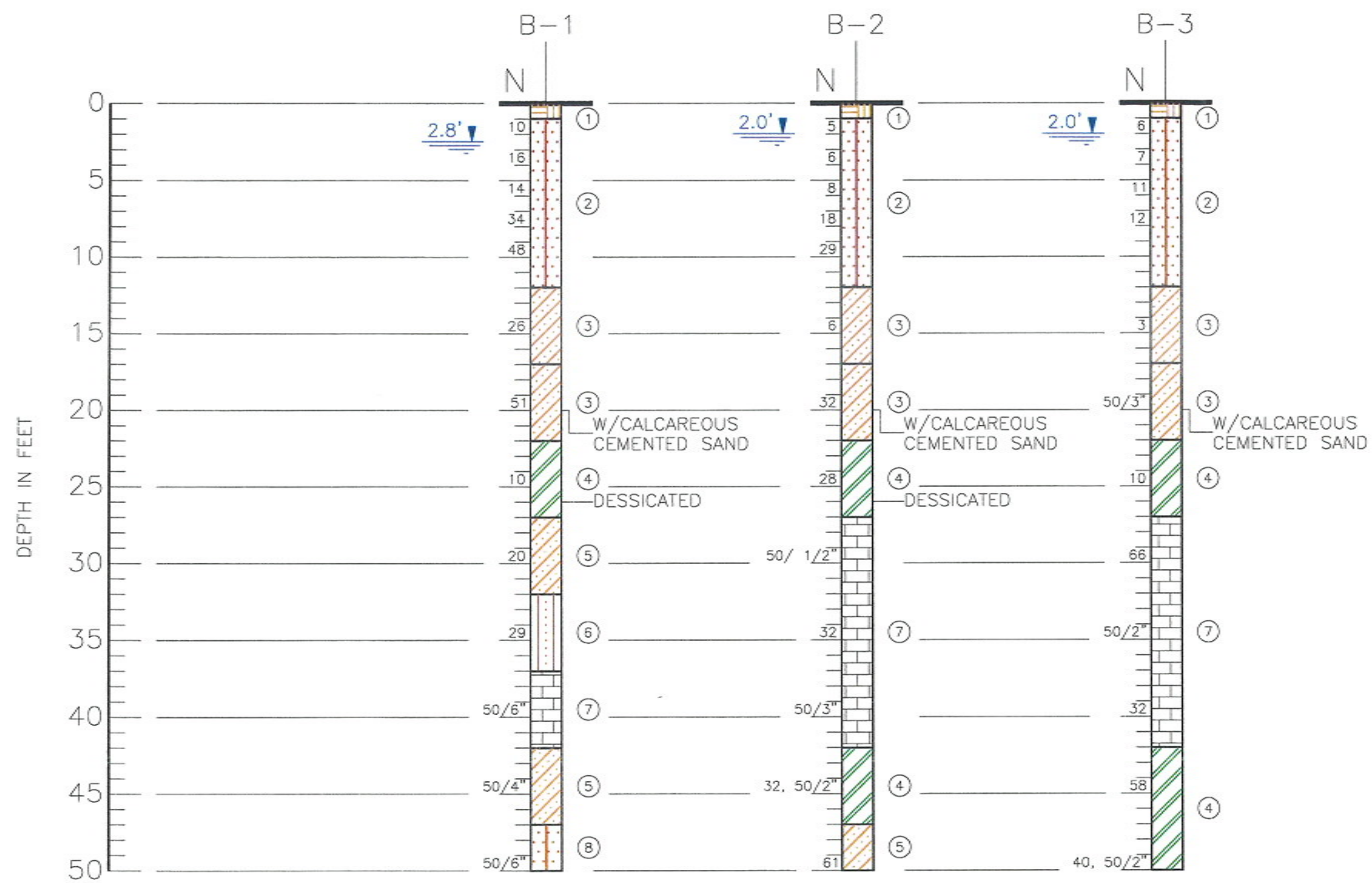
	FIGURE: 3



LEGEND

(1)	DARK BROWN SLIGHTLY SILTY FINE SAND (SP-SM) TOPSOIL
(2)	GRAY BROWN, REDDISH BROWN, SLIGHLTY SILTY FINE SAND (SP-SM)
(3)	LIGHT BROWN CLAYEY FINE SAND WITH PHOSPHATE (SC)
(4)	GRAY BROWN SILTY CLAY (CH)
(5)	LIGHT GRAY, GRAY CLAYEY FINE SAND (SC)
(6)	GRAY SILTY FINE SAND (SM)
(7)	LIGHT GRAY, TAN CALCAREOUS SILTY CEMENTED SAND (WEATHERED LIMESTONE)
(8)	GRAY SLIGHTLY SILTY FINE SAND (SP-SM)
(SP)	UNIFIED SOIL CLASSIFICATION GROUP SYMBOL AS DETERMINED BY VISUAL EXAMINATION
N	STANDARD PENETRATION RESISTANCE IN BLOWS PER FOOT
50/5"	(50) BLOWS REQUIRED TO DRIVE SAMPLING SPOON (5) INCHES
2.0'	DEPTH TO GROUNDWATER LEVEL IN FEET WITH DATE OF READING

SOIL BORING PROFILES
BAY PINES VA
RESEARCH BUILDING
PINELLAS COUNTY, FLORIDA

DRAWN: TMB	NODARSE & ASSOCIATES, INC.
CHKD: BMJ	
SCALE: NOTED	
DATE: 09-14-11	PROJ. NO: H4105204 — FIGURE: 4

Sep28, 2011-8:47am N:\Projects\2010\H4105204\Working Files\Diagrams-Drawings-Figures\H4105204-Figure-4.dwg